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FOUNDATION PROBLEMS OF THE EKLUTNA PROJECT

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FOUNDATION PROBLEMS OF THE EKLUTNA PROJECT

William R. Judd,[1] A.M. ASCE

SYNOPSIS

The Eklutna Project is a hydroelectric development being constructed by the Bureau of Reclamation near Anchorage in Western Alaska. The 830-foot power head is developed by tapping a natural lake 60 feet below its water surface with a 23,500-foot tunnel. An underground steel penstock leads to a powerplant with two 15,000-kw units. Although both were studied, a surface instead of an underground powerplant was selected. Ordinary geological explorations were complicated by rough terrain, extremely low temperatures, and an almost unknown geological situation. Drilling was done at the inlet area to the tunnel, a proposed dam site that later was eliminated due to foundation conditions, the tunnel portals, and the powerplant area. Construction of the inlet channel required the dredging of about 30 feet of unconsolidated rock flour with a very low cohesive strength, and underlying compact glacial till. The tunnel was driven through graywacke and argillite. High water flows and some squeezing ground hampered tunnel operations. As the result of load tests, point-bearing H-piles were selected for the powerhouse foundation. The major parts of the project now are complete and tunnel lining is in progress.

INTRODUCTION

The purpose of this article is to show the necessary integration of geology with planning, design, and construction of a major engineering project. On the Glenn Highway, 34 miles east of Anchorage, Alaska, the powerhouse, housing two 15,000-kw units, of the Eklutna Project, a hydroelectric development, is now in the final stage of construction. Generation of power from this plant in October 1954 will supplement the present supply of Anchorage. This latter supply is the old Eklutna 2,000-kw hydro-plant, the diesel generators in one-half of a tanker beached on Cook Inlet, and steam plants, partially or fully constructed.

The 830-foot power head in the new Eklutna plant is developed by tapping Eklutna Lake (Figure 1) about 60 feet below its surface with a 23,500-foot tunnel, a precast intake box (Figure 2), and 9-foot-diameter precast pipes (Figure 3). The tunnel leads to a penstock 1,056 feet long. Hydrology studies indicate sufficient supply from Eklutna Glacier to generate 137 million kwh of firm energy and 20 million kwh of nonfirm energy annually.

Natural Eklutna Lake, several years ago, was raised about 10 feet by a small concrete and earth dam constructed by the Anchorage Power and Light Company. Water is released into Eklutna Creek (Figure 4), travels to a

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forebay dam 50 feet high where it is diverted into an 1,800-foot tunnel for power development in the old Eklutna plant. Initially, the Bureau planned on increasing the lake level by raising the present dam. This would have permitted excavation of the tunnel inlet at or not more than 30 feet below the existing lake level. However, it proved more feasible to rehabilitate the present dam and excavate the tunnel inlet about 60 feet under the existing lake surface. The most efficient method of excavation was to drive the tunnel both directions from the bottom of a 210-foot-deep gate shaft. Another heading was driven from the outlet end of the tunnel.

Studies were made on the possibility of eliminating the tunnel by using a surface conduit down the slopes of Eklutna Creek; but the steepness and potential instability of high glacial till banks along this route precluded an economical development. Studies then were made on a tunnel and surface penstocks. However, as a result of exploratory studies, the penstocks were placed underground in a steeply inclined rock tunnel. From the powerplant, it was necessary to excavate a tailrace channel about 2,100 feet long to the Knik River at tide level.

TERRAIN SITUATION

Engineering geologic studies were started in the fall of 1950. The geologists had to unravel a complex geologic picture, traverse extremely steep and rugged terrain, and combat severe arctic weather. This combination resulted in an exploration program that had to be continued through one of the coldest winters on record in this area. It was necessary to complete the investigation and subsequent construction as soon as possible to alleviate the severe power shortage in a critical defense area. The terrain on the outlet end of the tunnel is densely wooded and has about a 1:1 slope. In addition, this slope is underlain by loose rock, and, therefore, continual care was required to prevent slides which could endanger the drilling equipment and men. At one point in the drilling, a small fire was started accidentally which denuded a few hundred yards of the slope. Loose rocks started rolling down the slope and temporarily put one drill out of commission.

Investigations at the inlet of the tunnel and the dam were hampered by the lack of trails, which all had to be swamped through very dense cottonwood trees and shrubs. Muskeg about 4 feet deep at the dam site required special drill platforms and extensive bulldozing before the drills could be moved into the area. Due to the extreme steepness of the terrain, as shown here, it never was practicable, due to the high cost, to place the drill rigs over the center portion of the tunnel. At the time it was doubtful if the information that could be so obtained would justify the cost of securing it. Furthermore, good surface reconnaissance would be of great assistance and would eliminate the necessity for many such drill holes.

These terrain difficulties were further complicated by a severe arctic winter. During January and February 1951, temperatures dropped as low as -55° F. For 2 weeks without cessation, the thermometer hovered between -20° and -50° F in the vicinity of the Lake and at the powerplant site. Wintertime daylight was only 3 to 4 hours long, from about 10 in the morning until about 2 in the afternoon. This lack of sunlight severely hampered surveying operations.

Surveying

The triangulation control was started in October 1950. Helicopters were used to place the surveyors on the stations at the top of surrounding peaks. But, due to the shortage of daylight, and severe weather, it was impossible to complete the control until early summer 1951. As standard plane-table topographic methods were found to be costly and time-consuming, a contract was issued for an aerial topographic survey. Poor weather again interfered with flying, and it was not until late fall 1951 that the final survey was completed.

SITE GEOLOGY

Eklutna Lake is in a deeply glaciated valley (Figure 4). The lake was formed by a natural dam left by a terminal moraine across Eklutna Creek. A thick layer of glacial till and rock flour underlies the lake and a considerable part of the shoreline. Apparently, the early flow of Eklutna Creek was somewhat to the north of the existing dam where it deposited large alluvial fans on the right abutment.

The tunnel penetrates the Chugach Range. During reconnaissance it appeared that rocks along the tunnel route were composed of a complex of argillites, graywackes, and an intrusive mass of greenstone. The argillite resembles a compact clayey shale, highly fractured, with numerous slickensides, and is very soft in some drill cores. The graywacke is a hard, greenish, fine-grained sandstone that has undergone some metamorphism and is severely fractured. The rock identified as greenstone is a hard, greenish, fine-grained material that appeared to be igneous in origin. In early surveys by the United States Geological Survey, this rock had been identified as an andesite (also an igneous rock). During excavation, neither of these igneous rock types were encountered in the tunnel. The entire mountain above the tunnel route is badly shattered by faulting, and numerous shear zones were observed on the ground, in airphotos, and from a helicopter.

The powerhouse site is underlain by glaciofluvial material which was deposited from the outwash of early advances of the Matanuska and St. George Glaciers. The depth of these deposits was unknown prior to drilling, but it was believed that bedrock would be quite deep. The rock in the hillside at the proposed penstock location is badly shattered and very unstable. Similar conditions prevail on nearby Pioneer Peak, where a massive rock slide occurred some 10 years ago. Since that time, small slides occur with such frequency that a telescope has been set up near the base of the mountain for tourists to observe the flying rocks on the mountainside some 5,000 feet above them.

Denudation of the penstock slopes by forest fires, severe winds, or artificial clearing would contribute to slope instability and also create a serious snowslide hazard. Therefore, the powerhouse was located away from the bottom of the slope. These same critical conditions would place a surface penstock in a hazardous situation. Furthermore, the badly fractured rock would provide poor foundations for the penstock anchors. In view of these conditions, the penstock was placed in a tunnel.

DRILLING

Inlet

At the proposed tunnel inlet, it was desirable to locate rock at shallow depths close to the lakeshore in order for as much as possible of the tunnel to be in rock (Figure 5). A number of holes were drilled along the shoreline for this purpose. These holes were supplemented by geophysical seismic explorations. Due to large boulders in the till and possibly due to the highly compact nature of the till, the seismic results did not give accurate bedrock depths, and, therefore, full dependence was placed on drill holes. These holes were drilled with a churn drill through the till until bedrock was reached; then a core drill was placed over the hole to drill into bedrock to invert grade. This drilling was very time-consuming due to the nature of the till and the severe weather. Although the drills were partially enclosed in wood shacks, when temperatures dropped below -40° F, it was necessary to suspend operations; neither men nor machines would work efficiently.

The glacial till was found to be composed of very compact rock flour with occasional sand strata which contained water. It was believed heavy support and possibly liner plate would be required in the till tunnel sections. Bedrock is a highly fractured graywacke-argillite complex that apparently would require only moderate support; although some difficulty might be encountered in tunneling through the argillite. On the basis of the geology, it was estimated that about 60 percent of the tunnel would require support; i.e., about one and one-half million pounds of steel supports and about 10 percent roof bolts would be used.

The probing of the subsurface at the tunnel inlet indicated a location known as the "B" line was the most economical tunnel site. On the "B" line, the rock was closer than at any other point along the lakeshore for the same length of tunnel. Rock outcropped at the shore about 3,000 feet upstream from this line, but a tunnel at that point would be longer and cost about one million dollars more than the "B" line tunnel.

Permafrost

During this stage of the drilling, it was necessary to determine if any part of the project would be in permafrost. This general area is close to the known southern boundary of permafrost in Alaska. Temperature readings with resistance thermometers were taken in each of several drill holes at the powerplant and the tunnel inlet areas. However, in all cases, the temperatures were from 32.1° to 39° F. The drill cores in the powerplant area did disclose, however, a few thin ice lenses.

Intake Channel

The character of the sediment underlying Eklutna Lake had to be determined as the dredged intake channel, about 900 feet long, would extend some 500 feet into the lake proper. There was about 3 feet of ice on the lake at this phase of the study and, thus, drill rigs could be located as desired on the lake. Undisturbed samples of the sediment, which varied from soupy to rubbery rock flour, could not be obtained. A drill casing placed upon this material sank under its own weight about 10 to 15 feet into the flour. However, by using a bailer, several jar samples were secured. Laboratory tests on the remolded material showed that the tan ϕ of the material was not measurable and the cohesion was one-half psi or less.

The hydrographic survey of the lake bottom indicated the sediment on the tunnel side of the lake had slopes of about 16:1. Drilling disclosed 30 feet of rock flour overlying a compact layer of glacial till with some sand strata and occasional boulders (Figure 6). Seismic results indicated bedrock at several hundred feet below the lake surface. As a result of these studies, the unconsolidated lake sediments, for specifications purposes, were classified: the rock flour was called "A" material, and the till was "B" material. It appeared that 20:1 would be the most economical stable slope for the soupy rock flour in the first 20 feet of "A" material; that 10:1 slopes would be used in the underlying rubbery rock flour; a berm would be left on top of the "B" material; and the sideslopes in the latter cut at 3:1. The intake box was located so that it would rest in and on glacial till with a minimum of 2 feet of gravel placed under and around it.

Dam Site Investigations

The location of the present dam was not deemed suitable for a higher structure due to the lack of adequate abutments and the cost of removing the old structure. Therefore, investigations for an earth dam were started about 100 feet downstream where a steep left abutment occurred and a fairly high right abutment could be obtained, although with some increase in the dam length (Figure 7). The left abutment had about 8 feet of muskeg overlying an unknown depth of glacial till. The right abutment was an alluvial fan overlying from 10 to 85 feet of very soft clayey silt which overlay an unknown depth of glacial till. (It is believed that the till is about 300 to 400 feet deep.) The till appeared to be sufficiently competent as a foundation for a moderately high dam; however, the soft clayey silt (later described as "bull liver") was extremely sensitive and incompetent. Laboratory studies showed that this layer would have to be removed.

Pervious materials were available in the alluvial fans on the right abutment. Riprap could be quarried from several large outcrops of graywacke upstream on the north side of the lake. However, good quality impervious material could not be found; the only available impervious material was the glacial till. This material was high in clay and water content (over the optimum moisture in most cases). It would have to be stockpiled and dried before placement. Previous experience indicated that after drying this material would become very hard and then would be difficult to break up and remoisten for embankment placement. The cost of obtaining suitable borrow, of excavating the soft silt under the right abutment, and of stripping the muskeg from the abutments could not be offset by the possibility of increasing the power head by the higher water surface provided by a higher dam. It proved more economical to lower the tunnel inlet 60 feet below the existing lake surface and only rehabilitate the existing dam.

Outlet Portal

By the time the dam drilling was completed, weather conditions permitted drilling to come out into the open. However, thawed ground made it very difficult to move the rigs from site to site. Drilling was started on the outlet of the tunnel and the surge tank site. This drilling verified earlier surface studies (Figure 8): The rock was mainly graywacke, but highly fractured and broken, with quartz and calcite stringers and numerous clay gouge seams. At the powerplant site, bedrock was 60 to 90 feet below the surface and was overlain by heterogeneous layers of silt, sand, boulders, clay, and about 6 feet

of muskeg at the surface. The water table was only a few feet below the surface. The graywacke bedrock was extremely fractured and contained numerous fault seams filled with gouge. In many of the holes, it was difficult to determine when the drill reached bedrock--core recovery, despite every precaution, was very low and ranged from 15 to 25 percent. A core longer than 1 inch could not be obtained.

Caissons, piles, spread footings, and a mat foundation all were considered as a plant foundation. The material was so irregular in stratification and compressibility and the difficulties inherent in unwatering a deep excavation seemed to preclude a mat foundation. For similar reasons spread footings appeared to be uneconomical. The high water table, occasional boulders, the possibility of "quick" materials, and the necessity for seating caissons deeply into the fractured bedrock all seemed to indicate very high costs for this type of foundation. Comparative estimates indicated that either point-bearing or friction piles would provide the most satisfactory and economical foundation. It was evident, however, that the complexity of foundation conditions would require extensive pile tests.

Underground Powerplant

Because of this questionable foundation, and because at this time considerable emphasis was given to the bomb protection of strategic installations, the possibility of an underground power-plant was studied. Although it was known that the rock was badly shattered, it was hoped that at a sufficient but economical depth within the mountain sound rock could be obtained for the chamber of an underground plant. Therefore, a drift was driven 375 feet into the hillside (Figure 8). Badly shattered graywacke, argillite, and extensive fault zones were encountered. There were a few areas of more sound rock, but they were too small in extent to accommodate an underground plant chamber. Furthermore, large water flows were encountered and the rock in the fault zones was so badly crushed that the light timber supports first used in the drift were destroyed by squeezing ground. It was found that the faults intersected any possible location of an underground chamber. With the assistance of these data, a comparative estimate was made between the cost of an underground and a surface plant. The underground plant apparently would cost about $500,000 more, despite the relatively costly foundation required for a surface plant. The Department of Defense then ruled that the additional cost for obtaining a bombproof plant could not be justified.

POWERPLANT TEST PILING

Investigations were continued on the surface powerplant. The bearing quality of the foundation was determined by driving test piles. Two timber piles were driven to determine the bearing capacity of the overburden materials. Load tests on these piles showed that, although they had met refusal during driving, apparently their points were underlain by compressible silts. Thus, the proposed powerhouse loads could not be taken on friction piles. Because about 400 piles would be required, point-bearing H-piles seemed to be the answer. However, as it was not possible to secure standard H sections, 90-pound rails were obtained from the Alaska Railroad Commission. Three of these rails were welded at their butts and the lengths spliced with fishplates. The driving records on these piles seemed to indicate that if driven to bedrock they would be suitable to bear the powerplant. Therefore, load

tests were instituted (Figure 9). The first few days gave satisfactory test readings, then after 17 days the piles started to settle, settlement then stopped for a few days, and then started again. This alternate settlement and rest continued over a 60-day period and the test was halted. The cause for this unusual test curve was not apparent because during driving every indication was that the pile was driven into bedrock. Although the rock was known to be badly fractured and faulted, this condition should not have caused the amount of settlement indicated in the test load readings. Therefore, an attempt was made to pull these 60- to 80-foot test piles. However, only portions of the piles could be retrieved. Apparently during driving the fishplates and rivets had sheared, the welds had broken, and, evidently, brooming had occurred.

Fortunately, by this time, it was possible to secure 14-inch, 73-pound H-piles from the tunnel contractor. Besides vertical loads, the foundation design had to take into account (1) numerous moderate to heavy earthquakes known to occur in the area, and (2) the horizontal thrust from the high-head penstock. For these reasons, piles on a 1:4 batter, as well as vertical piles, were used. These piles were driven and test loaded both vertically and laterally. Both the driving and loading records (Figure 9) were satisfactory and thus the foundation design incorporated point-bearing 14-inch, 73-pound H-piles.

The pile lengths were estimated from a bedrock contour map based on the drilling. This determination was very important, as every pile would have to be shipped from Seattle. As the shipping time was 2 to 3 months, sufficient piles had to be on the job at the time of driving to complete the job. The assumption was made, on the basis of the driving tests, that each pile would penetrate about 8 feet into the shattered bedrock. On this basis, 25,615 feet of piling were ordered.

To determine the possibility of pile deterioration, samples of the soil and ground water were examined for deleterious material. The soils had a high pH, and could be expected to react detrimentally on the piles. To prevent this corrosion, it was specified that all piles were to be coated with two coats of red lead prior to driving. To study the effectiveness of this coating, two test piles were driven adjacent to the powerhouse area where they could later be pulled. One of these piles is coated with red lead and the other is not. These piles are to be pulled about 10 years after construction and measured for corrosion effects.

Driving the powerplant piles started in September 1953, and by December all 289 vertical and 110 batter piles had been driven. Very few piles encountered boulders. The red-lead coating appeared to be undisturbed by the driving action. 18,138 feet (after cut-off) of piles were driven. This was less than the estimated amount because (1) the bedrock could be penetrated only 5 feet, and (2) considerably less damage from driving occurred on the upper end of the piles than had been expected as a result of observations on the test piles.

TUNNEL EXCAVATION

In October 1951, the contractor started excavation of the penstock shaft, the very difficult construction of access roads to the outlet portal, excavation of the penstock tunnel, and the construction of the dredge for the intake channel. The road construction was interrupted by numerous rock slides and, in

fact, was difficult to maintain during the entire construction period due to both rock and snowslides. The penstock construction proceeded normally, except in December 1952 the sheetpile bulkhead enclosing the open lower end of the penstock collapsed (Figure 10) as a result of hydrostatic pressure from excess flows from the main tunnel. At the gate shaft site continual mud flows in the glacial till around the upper end of the shaft (Figure 11) could not be controlled. As a result, the gate shaft superstructure was raised 15 feet. At the bottom of the shaft, tunnel excavation was started towards the lake and towards the main outlet. The glacial till section was driven with little difficulty, standard H-supports being used. Water flows from sand strata in the till never exceeded 300 gallons per minute. This section of tunnel was driven to Station 19+96 (Figure 5). At this point a temporary wooden bulkhead was placed and the tunnel concrete lined for some distance back towards the shaft. Then several feet back of this bulkhead, a steel bulkhead was set. Intake channel dredging then proceeded towards the tunnel until the wooden bulkhead was reached. Precast pipes then were fastened to a bell joint that had been formed in this end of the tunnel. In the meantime, tunnel driving to the north from the shaft was suspended because dredging operations required occasional blasting which might endanger the bulkheads and thus flood the tunnel. In fact, in October 1952, it was necessary to install a second steel bulkhead as excessive spalling was noted around the original steel bulkhead.

Dredging

The dredging operations encountered only minor difficulties. The accuracy of the slope stability analysis was verified by the fact that the final amount excavated, 394,000 cubic yards, was only about 15 percent more than estimated. Some of the glacial till required blasting as did some of the boulders in the till. (Several times the dredge operators insisted that they were cutting rock, although examination disclosed only glacial till.) The precast pipes were sunk (Figure 3) into their dredged channel and then divers fastened the sections together and to the precast intake box and to the tunnel. The intake box was placed by floating sections into place on a barge and then sinking them into the lake.

Outlet Adit

At the outlet end of the tunnel it was necessary to excavate an accessory adit to the west of the regular portal. Tunnel muck could not be dumped from the main portal due to the lack of space; from the accessory adit, the muck could be transported along the hillside to an adjacent draw. Generally, the rock in the outlet section was extremely fractured with several fault zones. Also, increasing water flows were encountered. By August 1952, these flows were about 3,000 gallons per minute. The water was cold, in fact, the average tunnel temperature during excavation was between 45^0 and 50^0 F (Figure 14). Considerable heavy supports were necessary near the outlet, but the excavation proceeded satisfactorily until Station 208+55 was reached on November 9, 1952.

A dry 2-foot gouge zone had been noted at Station 208+70. This zone started to emit water in ever increasing amounts until by November 8, 10 feet caved in and the flow was 3,000 gallons per minute (Figure 12). Then, at 9 p.m., on November 9, an estimated 18,000 gallons per minute surged from the zone, and complete squeezing and overbreaks estimated at over 100 feet high occurred (Figure 14). Men and machines had previously been evacuated. The flow

varied in intensity down to 7,800 gallons per minute, up to 13,000, and then down to 8,300 gallons per minute by December 5. This large flow could not be controlled when it first came out of the tunnel (Figure 13) and, thus, flooded the working area at the bottom of the hill. As a result, the sheet-piling around the penstock outlet collapsed (Figure 10), and the newly relocated road embankment was partially washed out.

During December, two 22-inch fan lines and appurtenant pumps were installed to handle the now diminishing water flow. Additional steel supports and solid lagging were placed where the tunnel had squeezed. By January 15, 1953, removal of the caved material was started, and by the end of the month, 2,000 cubic yards had been removed and the heading advanced into new ground. The water flow in the meantime diminished to 4,500 gallons per minute.

To accommodate any additional large flows, a channel in the bottom of the invert was partially excavated. The rock was blasted loose but was not to be removed until necessary. Tunnel operations now became fairly normal and the tunnel crews began to set driving records. On June 27, 1953, 76 feet in 24 hours were driven and 1,483 feet driven during the month. It still was necessary to occasionally support badly fractured zones, but additional water flows were limited and further squeezing ground was not encountered. One cave-in at the outlet portal did stop operations for a few days.

By February 1953, driving again started from the gate shaft heading. The rock was so badly fractured that it required continuous moderate to light support,[2] but fortunately the tunnel was fairly dry. By August 1953, the tunnel was almost holed through. The previous month had disclosed fractured but sound graywacke. It appeared likely that much of the proposed reinforcing steel in the lining could be eliminated. A consulting board recommended that only about one-third of the tunnel would require reinforcement, the final amount to depend upon the rock in the 3,500 feet of the tunnel not yet driven. The tunnel now was in a very dense, hard, green graywacke. Apparently this was the same rock that earlier had been identified on the surface as a greenstone. However, igneous rocks were not encountered; the full length of the tunnel, with the exception of the glacial till in the inlet area, was in graywacke and argillite. Occasional limestone boulders enclosed in the graywacke were found.

When the tunnel was holed through on October 15, 1953, a total of 1,659,000 pounds of steel supports had been used (Figure 14), only 10 percent more than had been estimated. Reinforcing steel was eliminated from about 15,000 feet of the lining with a resultant reduction in cost of about $800,000. At the junction of the headings, the line was off only 0.02 feet, the grade was off 0.12 feet, and the stationing off only 1.67 feet.

Surge Tank

The surge tank excavation proceeded pretty much according to schedule. The driving of the pilot raise was slow due to weathered and fractured rock with many mud seams. Overhead and wall bracing were necessary. This raise was holed through in May 1953 and final excavation to full diameter completed soon thereafter. The poor quality of the rock made necessary the use of continuous tight spiling behind all steel rings, and the rings were tied together to prevent downward movement.

2. Moderate support = 6-inch H-beams on about 5-foot centers; light support = 6-inch H-beams on about 6-foot to 8-foot centers.

Pressure Relief

One peculiar phenomenon was evidence of pressure relief in the argillite beds in the tunnel walls. This was particularly evident where the tunnel intersected the hanging wall. Apparently the excavation of the tunnel core resulted in relief of the large stresses in the surrounding beds. This relief of stress caused a very slow outward movement of argillite slabs on the hanging wall. The movement was not dangerous in that it did not create rockbursts; however, large openings occurred behind the moving slabs and occasional "wall falls" took place. Considerable grout will probably be required to adequately fill these openings.

WATER FLOW PREDICTIONS

The heavy water flows encountered in the early stages of tunnel excavation (Figure 14) had been predicted by surface geologic studies. These studies disclosed that (1) the topographic situation was such that the peak through which the tunnel was driven could be expected to act as a huge water reservoir; (2) the numerous shear zones and faults that intersected the tunnel lines were filled with tight clay gouge which could act as underground dams; and (3) there were several surface indications of ancient hot springs. The exploratory drift for the underground plant further verified the presence of large quantities of water in the mountain.

Therefore, although high water flows could be expected in the main tunnel, it was not expected that the center portions of the tunnel would be relatively dry. The exact reason for this lack of water can only be theorized. Apparently the mountain does act as a huge reservoir, but the numerous fault zones act as channels to carry the water away from the center core of the mountain; furthermore, the rock openings probably are so tight that large quantities of ground water cannot permeate to the tunnel invert which is about 5,000 feet under the summit of the mountain.

CONCLUSIONS

The planning and construction periods provided the engineering geologists with the following valuable conclusions:

(1) Unless an emergency exists, it is not economical to perform drilling operations in the winter in the Arctic;

(2) Detailed geological surface mapping, with petrographic identification of representative rock types, is necessary to the correct interpretation of geologic structure at tunnel level;

(3) And probably of the most importance, is the application of logistics to any exploratory operation in a remote area with a rigorous climate. Briefly, in this case, logistics consisted of getting the right equipment to the right place at the right time.

ACKNOWLEDGMENTS

The project was designed and constructed by the Bureau of Reclamation under the direction of L. N. McClellan, Assistant Commissioner and Chief Engineer. The construction engineers in charge for the Government were

Mr. Byron Felkner and Mr. L. F. Wylie. The project geologists were Mr. M. Athearn and Mr. C. E. McHuron. The tunnels and dam alterations were built by Palmer Constructors, a joint venture of A. L. Coker, Anchorage, and Peter Kiewit and Sons' of Omaha. The powerhouse construction was done by Rue Contracting Company of Fargo, North Dakota.

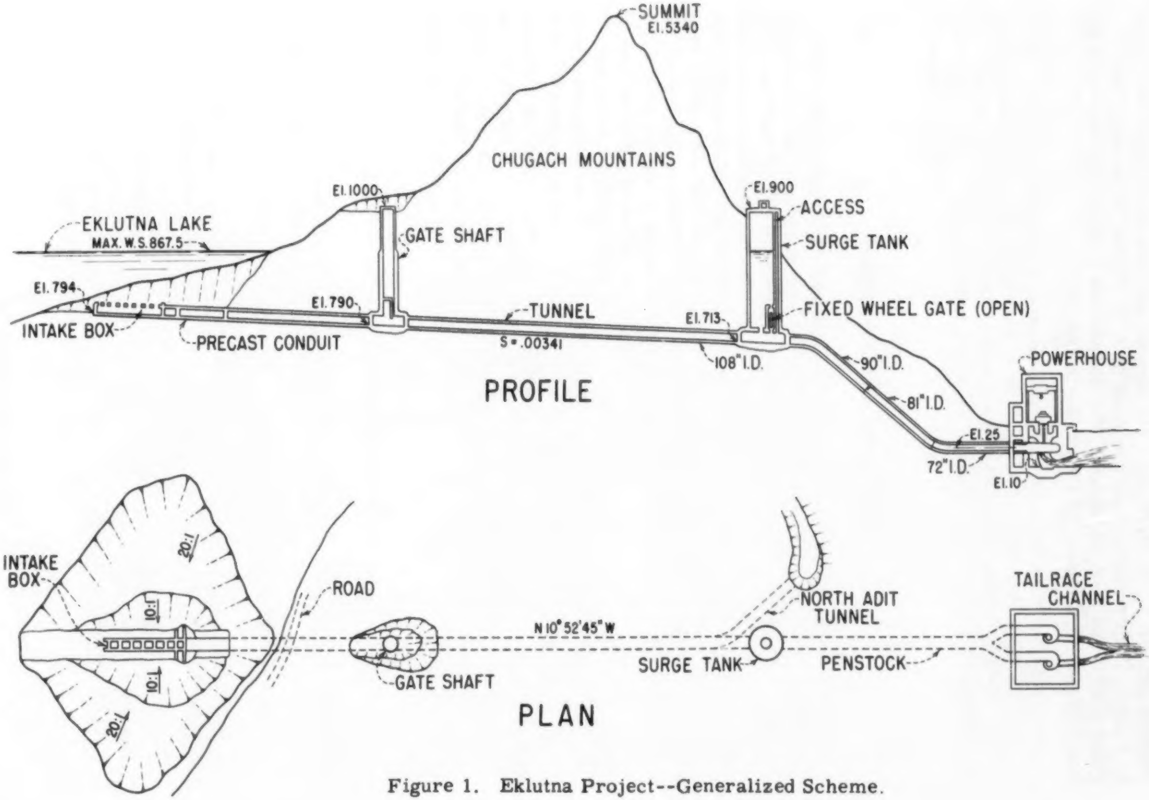

Figure 1. Eklutna Project--Generalized Scheme.

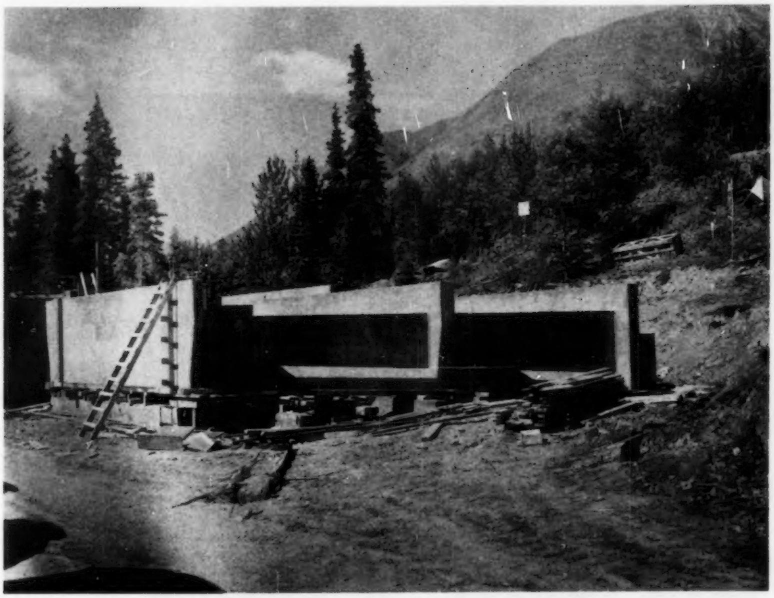

Figure 2. Precast Intake Box.

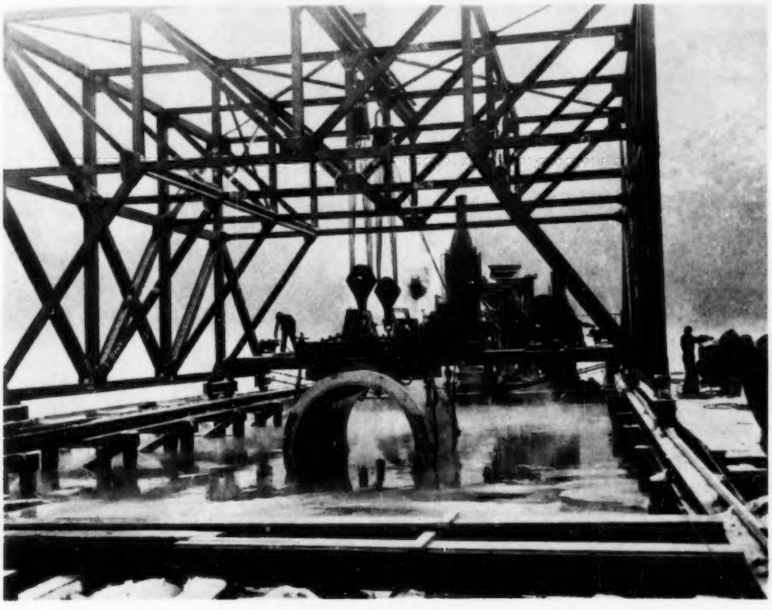

Figure 3. 9' Dia. Precast Pipe Being Placed.

Figure 4. Aerial Mosaic of Project.

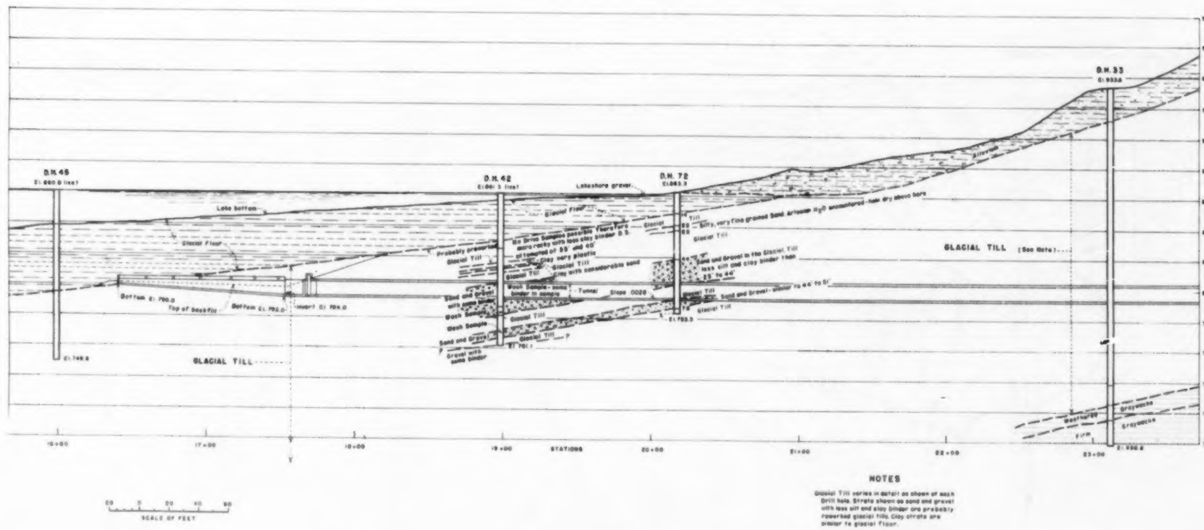

Figure 5. Intake Area--Geologic Section.

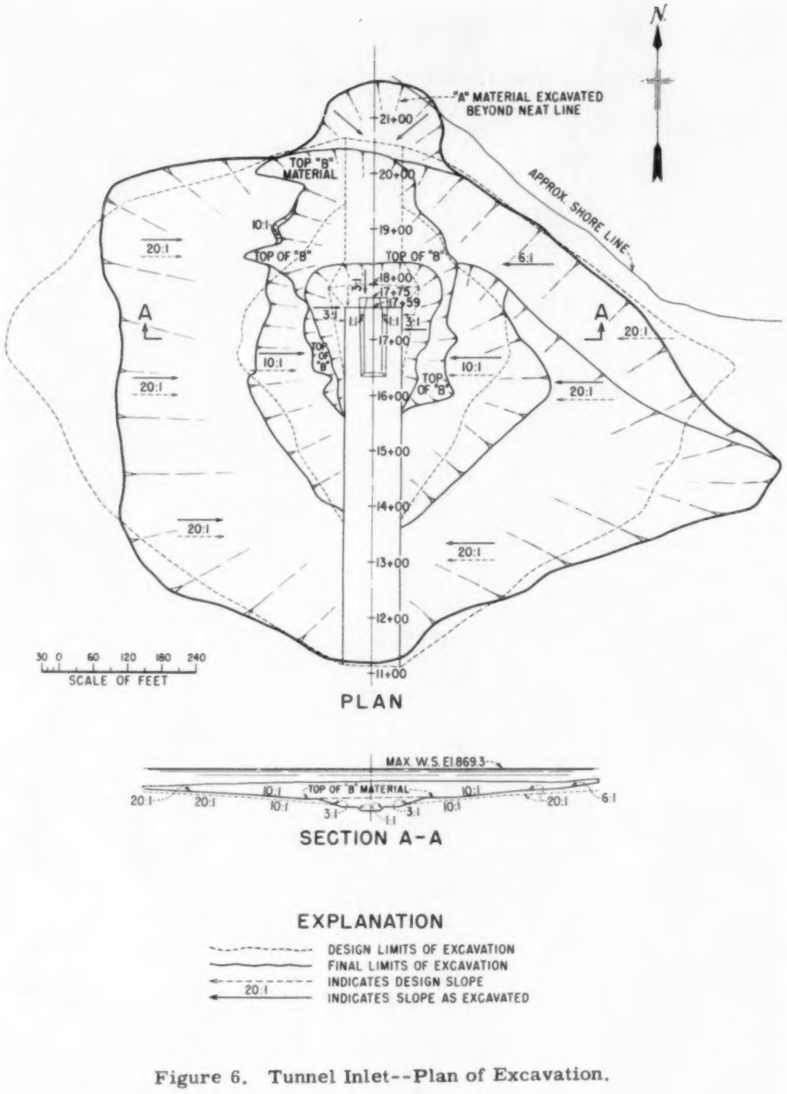

Figure 6. Tunnel Inlet--Plan of Excavation.

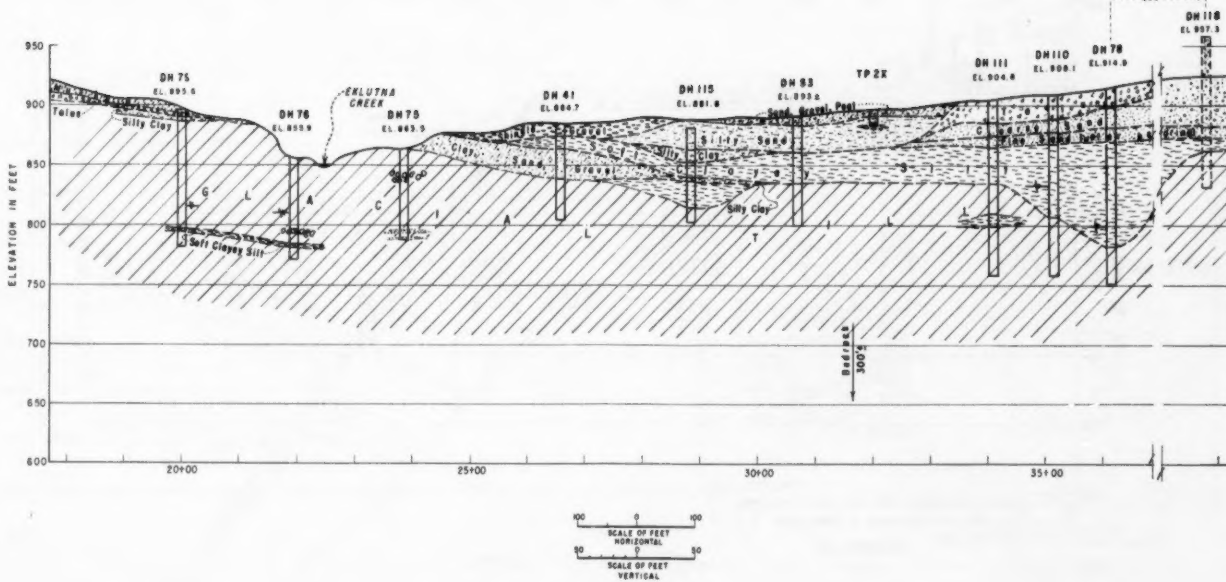

Figure 7. Dam Axis--Geologic Section.

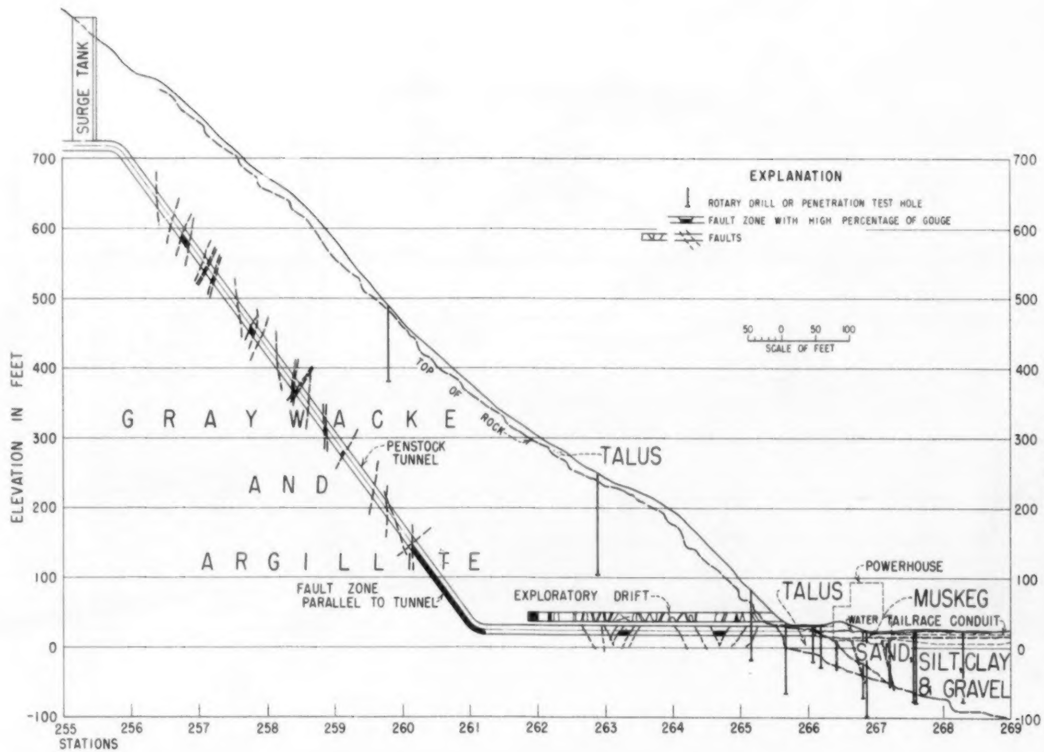

Figure 8. Outlet Area--Geologic Section.

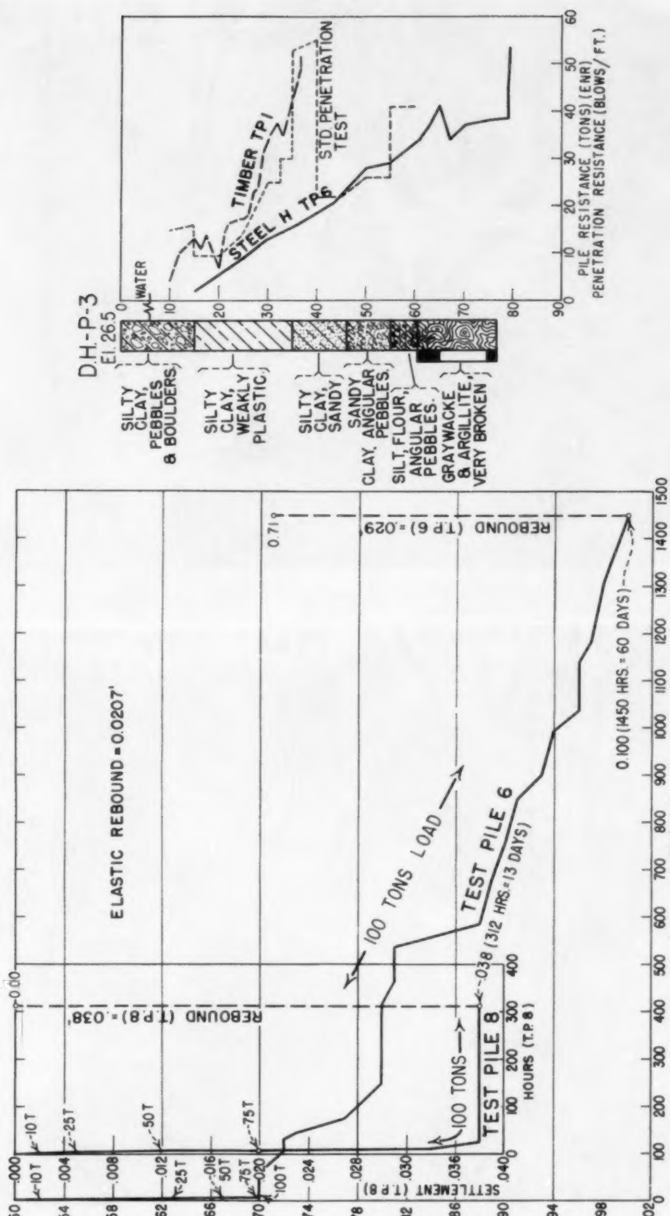

Figure 9. Test Piles--Loading and Driving Results.

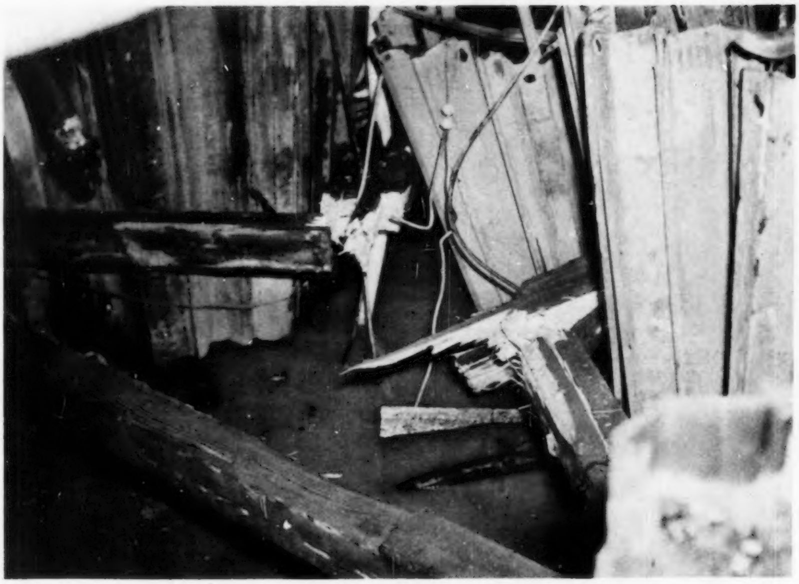

Figure 10. Collapse of Sheet Piles at Penstock Heading.

Figure 11. Mud Flows at Gate Shaft.

Figure 12. Tunnel Cavein near Sta. 208+70.

Figure 13. 18, 000 GPM Flow from Tunnel.

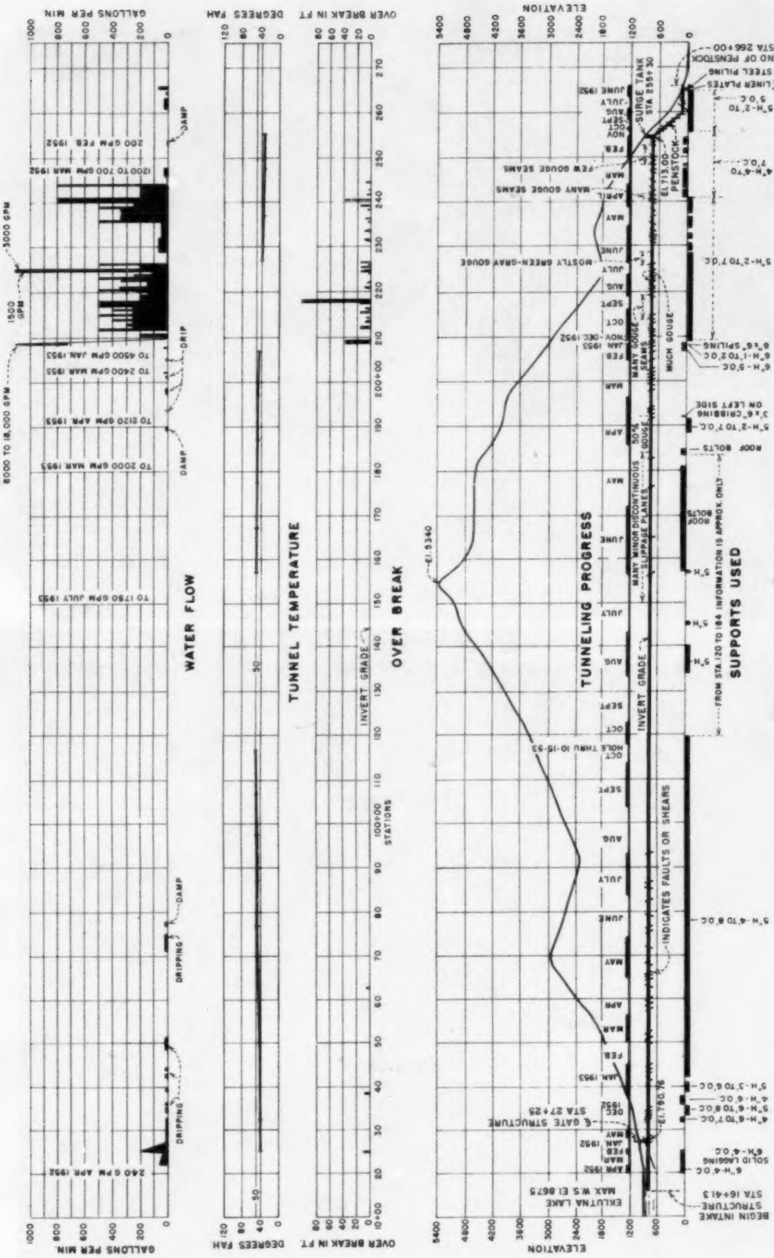

Figure 14 Summary